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EFFECT OF THE SERVICE LIFE ASSESSMENT APPROACH ON THE ENVIRONMENTAL BENEFIT OF USING SELF-HEALING CONCRETE IN MARINE ENVIRONMENTS

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ABSTRACT

To reduce concrete's susceptibility to cracking, full autonomous healing mechanisms are being studied today. A promising technique consists of incorporating encapsulated polyurethane-based healing agents. Upon crack occurrence, small capsules in close vicinity of the damaged area break. A polyurethane (PU) pre-polymer flows into the crack and after reaction with the surrounding moisture, the hardened PU prevents further accelerated ingress of aggressive substances at the original crack location. So far, promising results have mainly been obtained in proof-of-concept experiments. However, these tests do not allow for a proper estimation of the service life extension and environmental benefit that is possible when using concrete with self-healing properties. In this paper, such calculations have been performed for marine fly ash concrete with an in-house developed encapsulated high viscosity PU precursor. Since the service life calculation outcome very much depends on the probabilistic prediction model used and the underlying experimental input, two commonly used strategies were considered: one based on natural diffusion tests with chloride profiling at various exposure times and another based on accelerated chloride migration experiments at different ages. The first approach allows for a simultaneous fitting of the chloride diffusion coefficient, surface concentration and ageing exponent, while with the second one the chloride resistance after many years (time infinity) can be taken into account. Both approaches indicate a substantial prolongation of the service life when cracks, 300 µm in width, are healed autonomously, even if only partially. Nevertheless, the outcome of the two prediction methods is not the same (service life: 61-97 years versus 12-69 years, respectively). Depending on the applied approach, the required number of rehabilitation actions in time for the cracked (reference) concrete varies. As a consequence, service life related life cycle assessment performed in the SimaPro software clearly proves that the environmental benefits of the self-healing concrete will also differ for the ten baseline impact categories of the CML-IA impact method (56-74% versus 59-88%, respectively).

Keywords: Concrete cracking, Autonomous healing, Encapsulated polyurethane, Service life prediction, Chloride-induced steel depassivation, Life cycle assessment (LCA)

INTRODUCTION

In our modern society, more and more attention is going to sustainable design. As such, it is understandable that policymakers encourage also the building industry to develop innovative construction materials which are durable and environmentally friendly. Since concrete is one of the most widely used construction materials in the world [1], any new effort that could contribute to one of these two requirements for sustainability, is certainly welcome. The low tensile strength of concrete implies that the material is susceptible to cracking [2]. In steel reinforced concrete applications, corrosion inducing substances are able to penetrate faster via those cracks and reach the reinforcing steel. As a consequence, the concrete structure will require regular repair and thus additional concrete manufacturing over time to guarantee the ongoing fulfillment of its function. Therefore, it would be good that a mechanism could be introduced that would ensure autonomous crack healing upon crack occurrence, especially in the aggressive marine environments. Promising results have been obtained so far in mainly proof-of-concept experiments for concrete that contains brittle capsules filled with polyurethane-based healing agents [3-5]. Cracking triggers capsule breakage, release of the healing agent into the crack and crack closure upon reaction of the healing agent. However, despite the proven self-healing properties of this novel concrete type, the material is not commonly used yet in practice. One of the main reasons for this relates to the fact that the potential service life extension could not yet be quantified in an adequate and profound way. Without this quantitative proof, contractors are still reluctant to use the material on a large scale. First preliminary efforts have been done by Van den Heede et al. [6, 7] to provide this necessary proof by doing a probabilistic service life estimation based on chloride diffusion experiments in accordance with Visser et al. [8]. In this paper, that service life assessment approach has been slightly updated and compared with another one proposed by Visser [8-10], which is based on chloride migration experiments. The service life outcome of both approaches was used to determine a functional unit for life cycle assessment of the studied self-healing (SH) concrete and calculate its environmental benefits in comparison with traditional (cracked (CR)) concrete as such.

MATERIALS AND METHODS

Concrete mixture

The studied concrete mixture can be used in exposure class XS2 which corresponds with environments where concrete is permanently submerged in seawater. A fly ash (FA) containing concrete composition conforming to the k-value concept of NBN B15-001 was manufactured. By using the k-value concept, the maximum fly ash-to-binder (FA/B) ratio for a minimum total binder content equaled 15 %. Per m^3 of this concrete, this gives a CEM I 52.5 N content of 317.6 kg and a FA content of 56 kg for the binder fraction. A water content of 153 kg was assumed to achieve the required water-to-binder (W/B) ratio of 0.41 in compliance with the k-value concept. To make sure the concrete was sufficiently workable (slump class S3) some polycarboxylic ether-based superplasticizer needed to be added. The applied dosage was 3.0 ml/kg binder. Regarding the inert fraction, the concrete contained 696 kg river sand 0/4, 502 kg gravel 2/8 and 654 kg gravel 8/16 per m^3 of concrete. The 28-day compressive strength class was C40/50.

Self-healing mechanism

Some samples were given self-healing properties by incorporating cylindrical borosilicate glass capsules (inner diameter: 3.00 mm, outer diameter: 3.35 mm, length: 35 mm) filled with a one component PU precursor cf. Van Tittelboom et al. [3]. The capsules are characterized by a high brittleness. As such, they break easily upon crack occurrence. The healing agent was a non-commercial PU which was developed within the framework of another research project on self-healing concrete (SHEcon) [11]. It can be compared with a typical flexible PU foam of which the precursor essentially consists of methylene diphenyl diisocyanate (MDI) and a polyether polyol. This precursor reacts with water to create the foam that should heal the cracks. The moisture content of the concrete itself counts as the main water source.

Curing, sample preparation, cracking and healing

For the chloride migration tests, 18 cylindrical concrete samples (diameter: 100 mm, height: 50 mm) were cast in cylindrical PVC molds. Nine of them were used as they were, being uncracked. In the other nine, standardized cracks were created in accordance with Van den Heede et al. [6] by putting thin brass plates with a thickness of 0.3 mm at a depth of 25 mm in the molds just before casting. They were given self-healing properties as follows. The thin brass plates contained three holes with a 20 mm spacing. The glass capsules containing the PU precursor were put through these holes and fixed with nylon threads to the sides of the molds. After casting the concrete, capsule breakage and crack healing was triggered by pulling out the thin brass plates (cf. Van Belleghem et al. [12]). This was done after 18 days of optimal curing at 20°C and 95% RH. Since the exposure surface containing the artificially induced crack was a troweled surface with a certain roughness, it was mechanically flattened after the PU was given the time to harden completely for 48 hours. A layer of around 3 ± 1 mm was cut off this way. Afterwards, the cylinders were stored again under optimal curing conditions until the start of the chloride migration tests at 28 days.

Chloride diffusion & migration testing, self-healing efficiency

In previous research [6], (un)cracked and healed concrete have been subjected to chloride diffusion tests cf. NT Build 443. Chloride profiles were determined using potentiometric titration after 49 days of immersion in 165 g/l aqueous NaCl solution without pre-saturation in $Ca(OH)_2$. The effectiveness of crack healing was assessed by comparing the fitted profiles for uncracked, cracked and healed concrete. The profile of the self-healing concrete was quite similar to the one of the uncracked reference concrete. Yet, the profiles did not completely coincide, indicating that a 100 % healing performance could not be achieved so far. The profile of the self-healing concrete showed a near parallel offset with the one of the uncracked reference. This means that the fitted apparent diffusion coefficients for both types of concrete were quite similar ($D_{app\_self\,healing} = 0.87 \times D_{app\_uncracked}$) and that the fitted chloride surface concentration C_s and assumed initial chloride concentration C_0 of the former are somewhat higher than for the latter ($C_{s\_self\text{-}healing} = 1.13 \times C_{s\_uncracked}$, $C_{0\_self\text{-}healing} = 6.84 \times C_{0\_uncracked}$). Note that in case of the self-healing concrete, the observed 'initial' chloride content does not really correspond with the amount of chloride present in the concrete before start of immersion in the aqueous NaCl solution. It most probably represents the chloride ingress shortly after starting immersion due to capillary sorption phenomena in the crack that was not healed completely. The differences between the chloride profiles of the self-healing and uncracked concrete as observed in Van den Heede et al. [6] and as quantified above in terms of their difference in D_{app}, C_s and C_0 were taken into consideration when performing the comparative service life prediction of the concrete types further on (see Table 1). Van den Heede et al. [6] also conducted a series of natural diffusion tests in 33 g/l aqueous NaCl solution. Chloride profiling was done after 5 different immersion times. This approach resulted in the simultaneous fitting of the chloride surface concentration C_s, the instantaneous diffusion coefficient $D_{0,1}$ and a diffusion test based ageing exponent n_l cf. Visser et al. [8]. Their values amounted to 3.42 ± 0.05 m%/binder, 89 ± 3 mm^2/years and 0.33 ± 0.04, respectively.

The non-steady state migration coefficient ($\times 10^{-12}$ m^2/s) of the self-healing concrete after crack healing and the uncracked reference concrete was also experimentally determined. This was done in accordance with NT Build 492. The self-healing efficiency (SHE) for each tested specimen with encapsulated PU precursor was calculated by subtracting the chloride ingress from the crack tip from the average chloride ingress of the uncracked concrete and then dividing this value again by the latter value. The as such obtained SHE value was expressed in percent and should reveal whether the self-healing concrete really behaves as if uncracked (= a 100% self-healing efficiency). If not, the chloride migration coefficient of the self-healing concrete was calculated by assuming once the minimum, the mean and the maximum chloride ingress recorded in the vicinity of the crack that was only partially healed. For the service life prediction based on the non-steady state migration coefficients, a series of five chloride migration tests at different ages (7, 28, 91, 182 and 364 days) was also conducted on uncracked concrete cylinders to simultaneously determine the migration coefficient $D_{0,2}$ at the time of reference (28 d), the migration coefficient at time infinity D_{inf} (option A: zero value versus option B: non-zero value) and a migration test based ageing exponent n_2.

SERVICE LIFE PREDICTION

Chloride diffusion test based limit state function

The limit state function (Equation 1) used for estimating the time to steel depassivation is the same one that was applied in [6] and very similar to the one proposed by Visser et al. [8].

$$C_{crit} = C_0 + (C_s - C_0)\cdot\left[1-\mathrm{erf}\left(\frac{d}{\sqrt{4\cdot\exp\left(b_e\left(\frac{1}{T_{ref}}-\frac{1}{T_{real}}\right)\right)\cdot\frac{D_{0,1}}{1-n_1}\cdot\left(\frac{t_0}{t}\right)^{n1}\cdot t}}\right)\right] \qquad (1)$$

with, C_{crit}, critical chloride concentration (1.22 ± 0.02 m%/binder cf. [6]), C_0, the initial chloride concentration (m%/binder), C_s, the constant surface concentration (m%/binder), d, the concrete cover (50 ± 8 mm), b_e, a regression variable which follows a normal distribution (= 4800 ± 700 K), T_{ref}, the constant test temperature (= 293 K or 20 °C), T_{real}, the temperature of the seawater (= 283 K or 10 °C), $D_{0,1}$, reference instantaneous diffusion coefficient (mm²/years) at reference time t_0 which is usually 28 days (= 0.0767 years), n_1, the ageing exponent (–) and t, the exposure time (years).

Chloride migration test based limit state function

Visser et al. [8] also mentions an equation based on discrete exposure tests with only a brief exposure time. It includes the effect of concrete ageing (ageing exponent n_2) on the diffusion coefficient $D_{0,2}$. Since this equation is not based on diffusion tests involving continuous exposure for a long period, the whole time history of the changing diffusion coefficient is not an issue and division of $D_{0,1}$ by $(1 - n_1)$ as in the previous limit state function (Equation 1) is not required. Visser et al. [8] indicate that the discrete coefficient obtained from the brief rapid chloride migration can be used to represent the diffusion coefficient. However, in later publications [9, 10] Visser et al. emphasize that the use of an exponential ageing function implies that a zero migration coefficient is predicted for infinite age. This is not very realistic because field data obtained from structures of up to 41 years old were still characterized by non-zero long term migration coefficients. For this reason, the same authors proposed to introduce a migration coefficient at time infinity (D_{inf}). As such, the following chloride migration test based limit state function for service life prediction (Equation 2) was defined that was used further on this paper in comparison with the one based on the diffusion tests (Equation 1). Migration tests at multiple ages normally enable the simultaneous fitting of D_{inf}, $D_{0,2}$ and n_2.

$$C_{crit} = C_0 + (C_s - C_0)\cdot\left[1-\mathrm{erf}\left(\frac{d}{\sqrt{4\cdot\exp\left(b_e\left(\frac{1}{T_{ref}}-\frac{1}{T_{real}}\right)\right)\cdot\left(D_{inf}+\left(D_{0,2}-D_{inf}\right)\cdot\left(\frac{t_0}{t}\right)^{n2}\right)\cdot t}}\right)\right] \qquad (2)$$

Probabilistic calculation approach

Reliability indices (β) and probabilities of failure (P_f) were calculated using the First Order Reliability Method (FORM) available in the probabilistic Comrel software. Cf. fib Bulletin 34, these parameters need to meet the requirements for the depassivation limit state ($\beta \geq 1.3$ and $P_f \leq 0.10$) to qualify for use in a XS2 environment. Table 1 summarizes the applied distributions and their characterizing parameters of all model input. An additional calculation was performed for cracked concrete. This was done by assuming the model input of the uncracked concrete in combination with a concrete cover of only 25 mm instead of 50 mm to account for the presence of a 25 mm deep crack.

Table 1 – Input to the diffusion/migration test based limit state functions for service life prediction.

Variable (diffusion-based)	Distribution	Mean	Stdv.	Upper limit	Lower limit
$C_{0\_uncracked}$ (m%/binder)	Normal	0.06	0.01	–	–
$C_{0\_self\text{-}healing}$ (m%/binder)	Normal	0.41 (= 6.84×0.06)	0.01	–	–
d (mm)	Lognormal	50	8	–	–
b_e (K)	Normal	4800	700	–	–
T_{ref} (K)	Constant	293	–	–	–
T_{real} (K)	Normal	283	5	–	–
t_0 (years)	Constant	0.0767 (28 d)	–	–	–
$C_{s\_uncracked}$ (m%/binder)	Normal	3.42	0.05	–	–
$C_{s\_self\text{-}healing}$ (m%/binder)	Normal	3.86 (= 1.13×3.42)	0.05	–	–
$D_{0,1\_uncracked}$ (mm²/years)	Normal	89	3	–	–
$D_{0,1\_self\text{-}healing}$ (mm²/years)	Normal	77 (= 0.87×89)	3	–	–
n_1 (–)	Beta	0.33	0.04	0.00	1.00
C_{crit} (m%/binder)	Beta	1.22	0.02	0.00	2.00
Variable (migration-based)	**Distribution**	**Mean**	**Stdv.**	**Upper limit**	**Lower limit**
C_0 (m%/binder)	Normal	0.06	0.01	–	–
d (mm)	Lognormal	50	8	–	–
b_e (K)	Normal	4800	700	–	–
T_{ref} (K)	Constant	293	–	–	–
T_{real} (K)	Normal	283	5	–	–
t_0 (years)	Constant	0.0767 (28 d)	–	–	–
C_s (m%/binder)	Normal	3.42	0.05	–	–
D_{inf}_A(mm²/years)	Constant	0	–	–	–
D_{inf}_B(mm²/years)	Constant	30	–	–	–
$D_{0,2\_A\_uncracked}$ (mm²/years)	Normal	306	24	–	–
$D_{0,2\_A\_max\ self\text{-}healing}$ (mm²/years)	Normal	383	24	–	–
$D_{0,2\_A\_mean\ self\text{-}healing}$ (mm²/years)	Normal	490	24	–	–
$D_{0,2\_A\_min\ self\text{-}healing}$ (mm²/years)	Normal	597	24	–	–
$D_{0,2\_B\_uncracked}$ (mm²/years)	Normal	302	21	–	–
$D_{0,2\_B\_max\ self\text{-}healing}$ (mm²/years)	Normal	378	21	–	–
$D_{0,2\_B\_mean\ self\text{-}healing}$ (mm²/years)	Normal	483	21	–	–
$D_{0,2\_B\_min\ self\text{-}healing}$ (mm²/years)	Normal	589	21	–	–
n_2_A (–)	Beta	0.45	0.15	0.00	1.00
n_2_B (–)	Beta	0.50	0.12	0.00	1.00
C_{crit} (m%/binder)	Beta	1.22	0.02	0.00	2.00

LIFE CYCLE ASSESSMENT

Cf. ISO 14040, the LCA consisted of definition of goal and scope, inventory analysis, impact analysis and interpretation. The overall methodology was the same as in Van den Heede et al. for a steel reinforced concrete slab with a variable load of 5 kN/m^2 and a design service life of 100 years as functional unit (FU) [7]. As such, the extra material needed for repair in the course of time was considered. For a slab repair, an extra concrete volume representing the 50 mm cover on top of the rebars and an additional 16 mm was taken into account. For the self-healing concrete slabs and its repairs the presence of one layer of 4150 capsules filled with PU precursor in the tensile zone of the slab was considered. The life cycle inventory data related to the concrete, the rebars and the glass capsules were identical to those used in [7]. The data set for the PU precursor was slightly updated with newly obtained data regarding the density of the material. All LCA calculations were performed in SimaPro 8 using the CML-IA impact method with its ten baseline indicators (Table 3).

RESULTS AND DISCUSSION

Chloride migration test based self-healing efficiency

Figure 1 shows the chloride ingress as visualized with the silver nitrate colour indicator for each of the nine samples with self-healing properties. The other nine uncracked reference samples were assessed in the same manner, but due to the restricted length of this paper these graphs could not be included here. The uncracked concrete was characterized by a chloride ingress of 20 ± 3 mm on average. Some of the specimens with self-healing properties showed a very similar chloride ingress near the central area of the cross-section where the artificially induced crack was located. In other words, the chloride ingress of those samples (Figure 1: 1AA', 3AA', 5AA', 7AA') did not exceed the recorded crack depth. As such, they have a self-healing efficiency (SHE) of at least 100% and can therefore be catalogued as uncracked. Unfortunately, this was only the case for four out of nine samples. For the other five, the SHE value varied considerably from 22% to 76%. There are two main reasons for incomplete crack healing. Firstly, not all of the healing agent always flows out of the capsules upon breakage. Secondly, the mechanical flattening of the test surface before the migration test may have removed a crucial portion of the hardened PU and locally damaged the remaining PU filling in the crack. Preferably, alternative casting setups should be developed which do not require the mechanical flattening of the samples anymore before the migration test.

Estimation of D_{inf}

A plot of the measured non-steady state migration coefficients as a function of the five considered ages (7, 28, 91, 182 and 364 days) shows a clear exponential decrease in value (Figure 2). Using non-linear regression analysis, the estimated values for D_{inf}_A, $D_{0,2}$_A and n_2_A amounted to 0 mm^2/years, 306 mm^2/years and 0.45, respectively with an R^2 value of 0.99. A zero migration coefficient at time infinity was not really expected since the whole calculation procedure of Visser [10] has been developed specifically to avoid this kind of unrealistic material behavior in empirical modeling of the time dependent chloride ingress. The limited timespan that was considered for monitoring the migration coefficient for now is most probably the reason for this. More data at later ages (2 years, 3 years, etc.) are needed to allow for a more adequate estimation of D_{inf}. The required concrete cylinders to do those additional experiments have been made, but the appropriate testing ages for those specimens have not been reached yet. Nonetheless, the experimental data of Visser [10] indicate that the recorded migration coefficients after 3 years barely differ from those obtained after 1 year. Therefore, an additional fitting was done while taking into consideration the same value of the migration coefficient again after 3 years. The as such obtained values for D_{inf}_B, $D_{0,2}$_B and n_2_B are equal to 30 mm^2/years, 302 mm^2/years and 0.50, respectively. Both series of fitted values

(Figure 2: A & B) were implemented once in Equation 2 to see whether the inclusion of a limiting lower value of the migration coefficient has a significant effect on the service life or not.

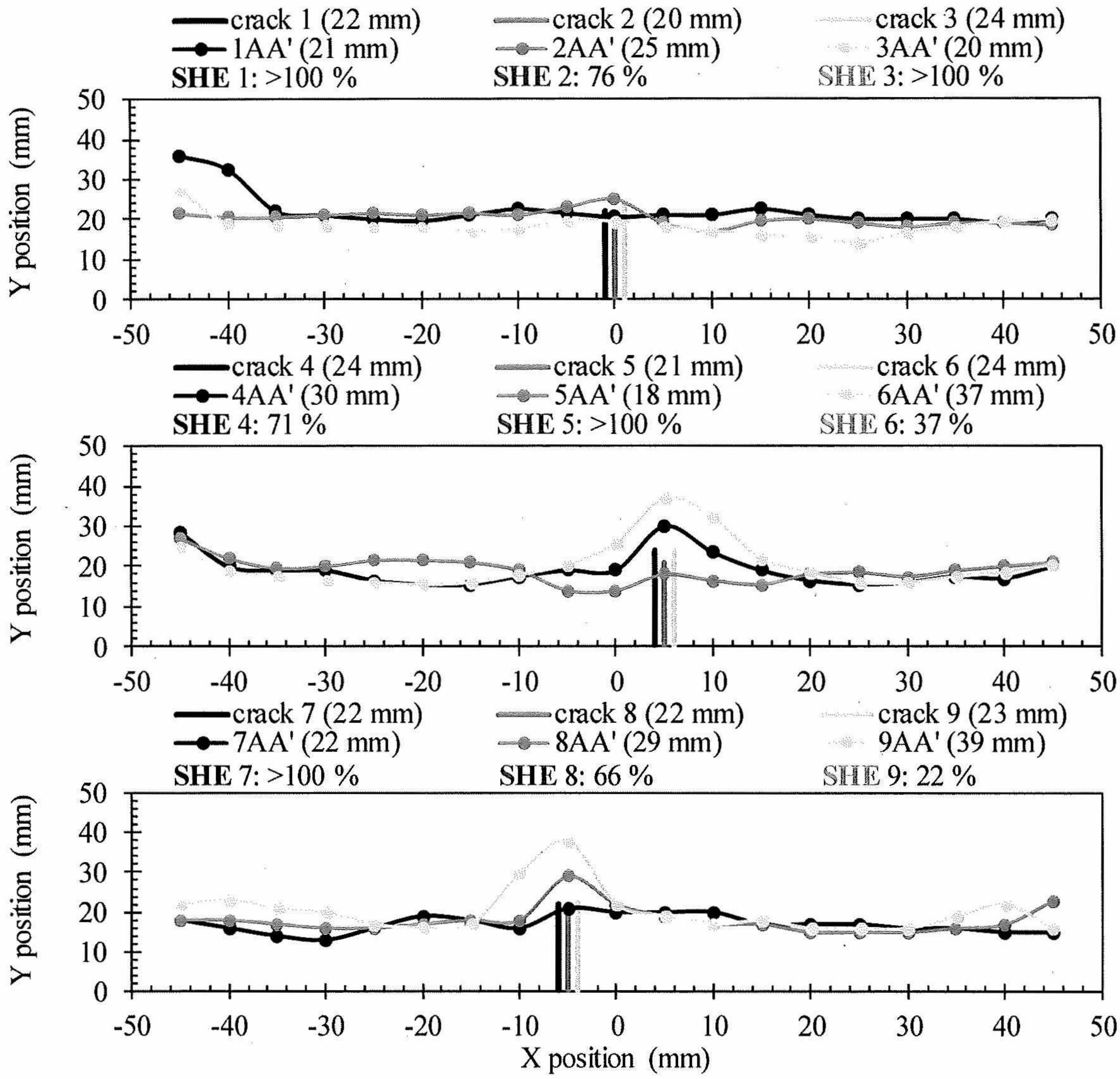

Figure 1 – Observed chloride ingress after migration tests cf. NT Build 492 on self-healing concrete with an encapsulated high viscosity PU precursor.

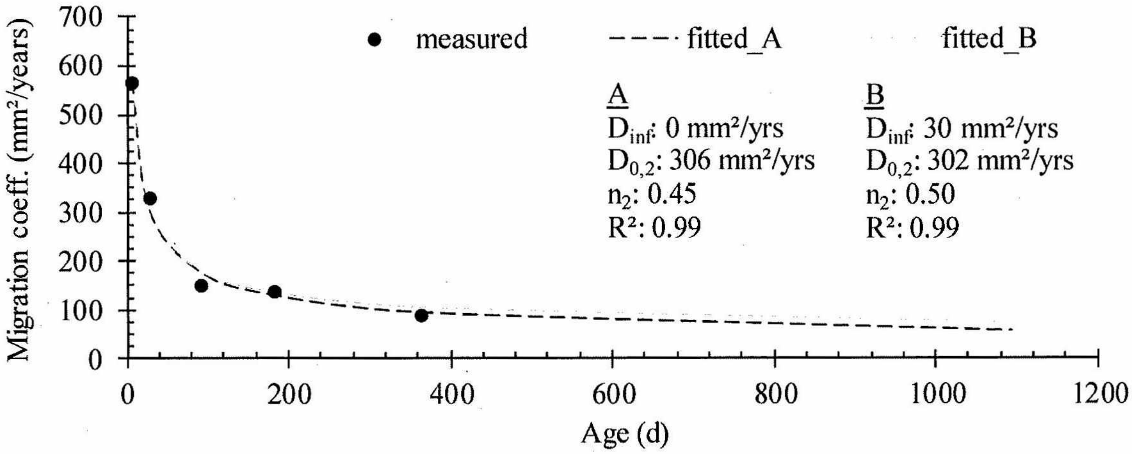

Figure 2 – Observed decrease in non-steady state migration coefficient (mm²/years) with age (d).

Service life prediction

As can be seen in Table 2, the service life prediction outcome of each of the considered concrete types is significantly different depending on the underlying test method (diffusion versus migration) and limit state function used. In general, a much shorter time to chloride-induced steel depassivation is expected when the service life assessment is based on migration tests instead of on diffusion tests. Thus, when interpreting the results of life cycle assessment calculations (see next section) for a service life related functional unit (e.g. a steel reinforced concrete slab with predefined mechanical load and service life), one should always check whether the underlying service life calculation approach is the same for all considered concrete types. With this condition fulfilled, the proportional differences in service life performance between concrete types are small. For example, the diffusion test based service life of 97 years for uncracked concrete (or self-healing concrete with a 100 % self-healing efficiency) as opposed to 7 years for cracked concrete indicates that the former performs around 14 times better than the latter. This factor amounts to a very comparable 15-17 when using the migration test approach. Note that both the diffusion and migration-based service life assessment indicate a very short service life of the slab implying a quite unfeasible required number of repairs.

Table 2 – Choride diffusion and migration test based service life prediction and repair frequency.

Diffusion-based	CR	SH	100% SH		
Service life (years)	7	61	97		
# repairs (–)	14 ×	1 ×	1 ×		
Migration-based	Cracked	SH			100% SH
	(A / B)	Min. (A / B)	Mean (A / B)	Max (A / B)	(A / B)
Service life (years)	2 / 4	12 / 24	15 / 33	22 / 48	30 / 69
# repairs (–)	49 × / 24 ×	8 × / 4 ×	6 × / 3 ×	4 × / 2 ×	3 × / 1 ×

Surprisingly, when looking at the effect of including the D_{inf} variable in the migration test based limit state function (A versus B), inclusion of a non-zero D_{inf} value seems to postpone the end of the initiation period of the corrosion process. This is simply the result of the fact that the other two variables in the D_{inf} related fitting process changed as well. The ageing exponent increased from 0.45 to 0.50. Given its mathematical position in the Equation 2, a slight increase in value of n_2 normally has a substantially positive impact on the service life prediction outcome. Moreover, the slight decrease of $D_{0,2}$ from 306 mm²/years to 302 mm²/years also contributed in this. Note that additional migration test results at later ages are still needed first to find further confirmation for these findings.

With respect to the performance of the self-healing concrete, one should keep the following in mind. Although partial crack healing already seems to result in a significant service life extension of 54 years (diffusion-based) or 13-29 years (migration-based), a 100 % self-healing efficiency corresponding with uncracked state still has the preference as service life would go up with no less than 90 years or 28-65 years, respectively. Thus, additional research efforts are still needed to further optimize the self-healing concrete with the encapsulated high viscosity PU precursor to reduce its variability in healing efficiency and to achieve a SHE value of 100 % at all time.

Life cycle assessment

Given chloride diffusion test results and the corresponding service life prediction, the ten baseline impact indicators of the CML-IA impact method related to the self-healing concrete are no less than 56-74 % lower than the ones associated with ordinary cracked concrete in submerged marine environments (Table 3). This can mainly be attributed to the fact that regular repair works (every 7 years, 14 times) are needed for the cracked concrete within a 100-year time span, while for self-healing concrete with intermediate self-healing efficiency this is only required once after around 61 years. A 100% self-healing efficiency would result in the same environmental benefits because an estimated service life of 97 years still implies one repair within the predefined 100-year lifespan of the slab. With chloride migration experiments as basis for service life prediction, the environmental benefits of using self-healing concrete seem even more pronounced. Depending on whether the minimum, mean or maximum partial healing performance was considered, the environmental benefits are in the range of 59-77%, 63-81% or 66-85%, respectively. In case of a SHE value of 100%, the environmental burdens would be reduced with no less than 70-88% in comparison with ordinary cracked concrete. Again, the main reason for this behavior relates to the fact that considerably less additional concrete needs to be manufactured over time to repair the self-healing concrete slabs. As such, the minimal impacts associated with the incorporation of one layer of 4150 capsules filled with the PU precursor within the tensile zone of the slab, are easily overcome.

Table 3 – Environmental impact for the cracked (CR) and healed (SH) steel reinforced concrete slab (load: 5 kN/m², service life: 100 years).

Impact category indicator	CR, diff.	SH, diff.	100% SH, diff.
Abiotic depletion potential (ADP, × 10^2 MJ fossil fuels)	0.7	– 66%	– 66%
Global warming potential (GWP, × 10^3 kg CO_2 eq)	1.7	– 74%	– 74%
Ozone depletion potential (ODP, × 10^{-4} kg CFC-11 eq)	0.5	– 73%	– 73%
Human toxicity potential (HTP, × 10^2 kg 1,4-DB eq)	1.4	– 69%	– 69%
Freshwater aq. ecotoxicity potential (FAETP, × 10^2 kg 1,4-DB eq)	0.4	– 61%	– 61%
Marine aq. ecotoxicity potential (MAETP, × 10^6 kg 1,4-DB eq)	0.4	– 65%	– 65%
Terrestrial ecotoxicity potential (TETP, kg 1,4-DB eq)	0.5	– 70%	– 70%
Photochem. ozone creation potential (POCP, × 10^{-1} kg C_2H_4 eq)	1.5	– 56%	– 56%
Acidification potential (AP, kg SO_2 eq)	3.1	– 71%	– 71%
Eutrophication potential (EP, kg PO_4 eq)	0.9	– 64%	– 64%

Impact category indicator	CR, migr., A / B	Min. SH, migr., A / B	Mean SH, migr., A / B	Max. SH, migr., A / B	100% SH, migr., A / B
ADP (× 10^2 MJ fossil fuels)	2.0 / 1.1	– 75% / – 67%	– 79% / – 71%	– 82% / – 74%	– 84% / – 78%
GWP (× 10^3 kg CO_2 eq)	5.0 / 2.6	– 77% / – 72%	– 81% / – 76%	– 86% / – 80%	– 88% / – 84%
ODP (× 10^{-4} kg CFC-11 eq)	1.6 / 0.8	– 77% / – 72%	– 81% / – 75%	– 85% / – 79%	– 87% / – 83%
HTP (× 10^2 kg 1,4-DB eq)	3.9 / 2.1	– 75% / – 68%	– 79% / – 72%	– 83% / – 76%	– 85% / – 80%
FAETP (× 10^2 kg 1,4-DB eq)	1.2 / 0.6	– 72% / – 63%	– 76% / – 67%	– 80% / – 70%	– 82% / – 74%
MAETP (× 10^6 kg 1,4-DB eq)	1.0 / 0.5	– 74% / – 66%	– 78% / – 70%	– 82% / – 73%	– 84% / – 77%
TETP (kg 1,4-DB eq)	1.5 / 0.8	– 75% / – 69%	– 79% / – 73%	– 84% / – 77%	– 86% / – 81%
POCP (× 10^{-1} kg C_2H_4 eq)	3.7 / 2.1	– 69% / – 59%	– 73% / – 63%	– 77% / – 66%	– 79% / – 70%
AP (kg SO_2 eq)	9.1 / 4.8	– 75% / – 69%	– 80% / – 73%	– 84% / – 77%	– 86% / – 81%

EP (kg PO_4 eq)	2.6 / 1.4	– 73% / – 65%	– 77% / – 69%	– 81% / – 72%	– 83% / – 76%

CONCLUSIONS

Probabilistic service life estimation based on chloride diffusion and migration tests both indicate a substantial extension in service life when using self-healing concrete with an encapsulated high viscosity PU precursor for steel reinforced concrete slabs located in submerged marine environments. With chloride diffusion test results as model input, the service life of the self-healing concrete would amount to 61-97 years, while this would only be 7 years in case of ordinary cracked concrete. With chloride migration test results as model input, the concrete with self-healing properties would last for 12-69 years, which is still much longer than the 2-4 years of service life for the cracked concrete. As such, the ten baseline impact indicators of the CML-IA LCA impact method indicate substantial environmental benefits for self-healing steel concrete slabs (load: 5 kN/m^2, design service life: 100 years) of 56-74% (based on diffusion tests) or 59-88% (based on migration tests). It further demonstrates the high sustainability potential of this novel material.

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